

# Study and Analysis of Intz water tank with manual and software-based design with base isolation

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**Abstract**— The seismic response of an overhead water tank, cylindrical, extra-large water storage tank by using triple friction pendulum system is analyzed. Most of the overhead tanks have a fundamental frequency which includes a series of resonance of greatest earthquake ground motions. It is an operative way to reduce the response of an isolation system used for storage of water tanks under a sturdy earthquake. However, it is problematic to implement in preparation with common isolation bearings

The research is directed with study of existing studies in the field of seismic behavior of intz water tank. Base isolation is one of the technologies applied to decrease the consequence of earthquake effect. The principle is to separate the base of the overhead water tank from footing ground. The problematic is taken as Intz water tank design to survive water tank against seismic accomplishment. three categories of base are used to analyses and compare overhead first is manual design of intz water tank with fixed base + response by SRSS and second case is intz water tank with fixed base by sap2000 and Third case with is intz water tank with triple friction pendulum on sap2000.

The software SAP 2000 are used to assessment fixed and triple friction pendulum base intz water tank. It is primary period in India when overhead water tank is tested with triple friction pendulum isolation are analyzed for seismic zone V. It is initiate from results that deflection and base shear analyzed with triple friction pendulum are lesser than fixed base with outstanding margin and it is determined that study endorses use of triple friction pendulum base isolation for seismic zone V in India.

**Keywords**— Intz water tank, Seismic, Fixed Support, triple friction pendulum Support, SAP2000, Deflection, base shear.

## I. INTRODUCTION

Elevated Water tanks are salvation developments which are being constructed in growing numbers to store water for drinking purpose. The capacities of these containers are huge and have capacities of around 1800 m<sup>3</sup> or it may be large depends on the population of that area. Elevated water tank consists of an RCC container or it may made up of steel tank, which contains the large amount of water, the seismic study of these structures is a difficult and thought-provoking task because the construction of the tank based on the with a smaller number of footing and column as compare to building and the soil erection contact must be considered. water tanks present an excessive risk if they were failed during an earthquake. Base isolation is a demonstrated knowledge for the seismic strategy of structures. The system diminishes the probability of structural and non-structural damage to a water tank exposed to seismic forces. By using base isolation, we can reduce the lateral forces and displacement of the structure which can damage the structure through earthquake. Due to which we can save the government property and water which get distributed to the people.

However, in spite of base isolation's safety benefits, the technology is under operated. Although tall, flexible, and non-critical facilities such as office buildings are not the most ideal candidates for base isolation, they may still achieve an optimal seismic design by using the technology. Therefore, in order to increase the quality and prevalence of base isolated structures, there is a need to study the technology's seismic performance enhancements and cost effectiveness for projects on which the system is infrequently used. IS 1893:2002 is the code to design structures under earthquake zones. There are two major methods of seismic analysis which are

**1. Response Spectrum Analysis:** This Analysis is based on ideal predefined statistics which are not actual time data collected since actual earthquake in the part.

a) SRSS b) CQC

**2. Time History Analysis:** This Analysis is based on genuine real time data composed under actual earthquake. Elevated water tank response and behavior is composed in real time and can be used to design future elevated water tank under seismic loading.

#### Need of study

1. Baseisolation technique is newly isolated structure which is provided at the base of structure.it is only performed on building and hospital etc. None of the research is done on base isolation on elevated water tank. 2.So the approach is done on Base isolation for Elevated water tank or manual and software comparison.

## II. PROBLEM DESIGN

The design of overhead Intze Water Tank is carried out using the manual and computer aided design software sap2000 Elevated storage reservoir. The design is carried out as per relevant analysis procedures combined with Indian Standard Codes of Practices. The water tank dome is designed by working Stress's method. The foundation forces at the level of safe bearing capacity are also evaluated The software also gives the shape description of the tank and keeping various constraints, one can change the governing constraint to get the optimum result and safe design with economy.

**PLAN DATA:** Structural design of intz water tank of capacity 900000 liters.

**Location of site:** BHUJ (GUJARAT)

Type of tank: Intze water tank

Staging System Chosen: Column Braced

#### Geometrical Data

Seismic Zone: V

#### Soil properties

Soil Description: Medium soil

Safe Bearing Capacity at Depth 1 m: 150 Kn/m<sup>2</sup>

#### Manual Design of intze water tank :-

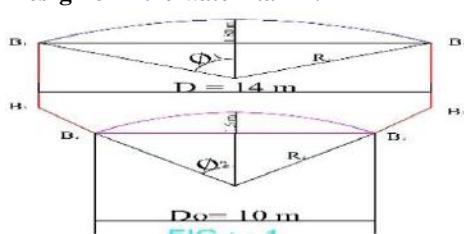


Fig. 1: Dimension of intz tank

Volume of water Tank = 900000.00 litre capacity

Height of Staging =16.00 m

Suppose the Diameter of Cylindrical's portion = D = 14.00 m

And Radius of Cylindrical's Portion R = 7.00 m

Suppose the Diameter of Ring Beam B2 = Do = 10.00 m

And Radius of Ring Beam B2 = Ro = 5.00 m

Suppose Height h<sub>o</sub> of Conical Dome = 2.00 m

[www.ijaers.com](http://www.ijaers.com)

Suppose Rise h1=1.800 m ; Rise h2 = 1.600 m

The Radius R2 of lowest dome is given by

$$h2 * (2 * R2 - h2) = R2^2$$

$$1.6 * (2 * R2 - 1.6) = 5^2$$

$$\text{Radius of lowest dome } R2=8.610 \text{ m}$$

$$\sin\theta_2 = 5/8.61 = \theta_2 = 35.500^\circ$$

$$\cos\theta_2 = 0.8141 \tan\theta_2 = 0.71330$$

suppose h be the height of cylindrical's portion

From which h = 4.780 m

Permitting for free board keep h = 5.00 m

For the top dome, the Radius R1 is given by

$$h1 * (2 * R1 - h1) = R1^2$$

$$1.600 * (2 * R1 - 1.600) = 7.00^2$$

$$\text{Radius of lowest dome } R1=14.510 \text{ m}$$

$$\sin\theta_1 = 5.00/14.510 = 0.48240$$

$$\theta_1 = 28.840^\circ$$

$$\cos\theta_1 = 0.87600$$

#### 2 - DESIGN OF TOPMOST DOME

Suppose Thickness t1 = 100.00 mm

Taking Live load = 1500.00 N/m<sup>2</sup>

Total P per sq.m of dome = 0.1 x 25000.00 + 1500.00

$$P_{total} = 4000.00 \text{ N/m}^2$$

Meridional's Thrust's at edges

$$T1 = P * R1 / 1 + \cos\theta_1$$

$$T1 = 4000.00 * 14.510 / 1.00 + 0.8760 = 30938.00 \text{ N/m}$$

Meridonal's Stress's per metre = T1/t x d

Meridonal's Stress's per metre = 30938.00 / 100.00 x 1000.00

Meridonal's Stress's = 0.31 N/mm<sup>2</sup> :- safe

Extremes hoops Stress's arises at the centre and its magnitude's = P \* R1 / t1 x 2

$$= 4000.00 * 14.510 / 2.00 x 0.10 = 290200.00 \text{ N/m}^2$$

**Extremes hoops Stress's = 00.29 N/mm<sup>2</sup> :- safe**

Since's the Stress's are in safe limit, offer's nominal reinforcements @ 0.3 %

$$A_s = 0.30 x 100.00 x 1000.00 / 100.00$$

$$A_s = 300.00 \text{ mm}^2$$

Using 8 mm Ø bar , AØ = 50.00 mm<sup>2</sup>

$$\text{Space} = 1000.00 x 50.00 / 300.00 = 160.00 \text{ mm}$$

**:- 8.00 mm Ø bar @ 160.00 mm c/c in both direction**

#### 3 DESIGN OF TOPMOST RING BEAM B1

Horizontal Element of T1 is given by

$$P1 = T1 * \cos\theta_1$$

$$P1 = 30938.00 x 0.8760 = 27102.00 \text{ N/m}$$

Whole tension's tending's to ruptures the beams = P1 x D/2

$$A_c = 27102.00 x 14.00 / 2.00 = 189712.00 \text{ N}$$

Whole's tensions tendings to rupture's the beam = 189712.00 N

Permissible Stress's in HYSD bars = 150.00 mm<sup>2</sup>[ IS 456 :2000]

A<sub>sh</sub> = Whole tension tending to rupture the beam/

Permissible Stress's hysd bars

$$A_{sh} = 189712.00 / 150.00 = 1265.00 \text{ mm}^2$$

Ash actual = 1265.00 mm<sup>2</sup>

No. of 20.00 mm Ø bars = Ash actual / Area of bar

No. of 20.00 mm Ø bars = 1265.00 / 314.160

No. of 20.00 mm Ø bars = 4.00

Actual Ash offered = 314.160 x 4.00 = 1257.00 mm<sup>2</sup>

The areas of cross sections of rings beams is given by

Ac = Whole tension tending to rupture the beam / A + (m-1)

x Ashp

= 189712.00 / A + (13.0-1.0) x 1257.00 = A = 143014.00 mm<sup>2</sup>

Offer ring beam of 360.0 mm depth and 400 mm width. Tie the 20 mm Ø rings 6.00 mm Ø Nominal stirrups @ 200.00 mm c/c

Topmost Ring Beam = 400.0 x 360.0 = 144000.0 mm<sup>2</sup> :-

**Safe**

#### **4 DESIGN OF CYLINDRICAL's WALL's**

In the membrane analysis, the tank is projected to be free at top and bottom. Extreme hoop's tension's occur's at the base of wall, its magnitude is given by :-

$$P = w * h * D / 2$$

$$P = 9800.00 * 5.00 * 14.00 / 2.0$$

$$P = 34300.00 \text{ N/m}$$

Area of Steel Ash = P / Permissible Stress's

$$\text{Ash} = 34300.00 / 150.00$$

**Ash = 2286.00 mm<sup>2</sup> per metre height**

Provided that ring's on both the faces,

Ash on each-face = 2286.00/2.00

Ash on each-face = 1143.00 mm<sup>2</sup>

Space of 12 mm Ø rings @ per m = 1000.00 x 113.00 / 1143.00 = 98.90 mm

Offer 12 mm Ø rings @ 95 mm c/c at bottom. This space can be increased at the top

Actual Ash offered = 1000.00 x 113.00 / 95.00

Actual Ash offered = 1190.00 mm<sup>2</sup> on each-face

Permitting 1.200 N/mm<sup>2</sup> Stress's on composite section

1.2 = At / 1000.00 x t + (m-1) Ash offered x 2.00

1.2 = 343000.00 / 1000.00 x t + (13.0-1.0) x 1190.00 x 2.00

$$t = 257.330 \text{ mm}$$

minimum thickness = 3H' + 5.00 = 3x5.00 + 5.00 = 200.00 mm

$$\text{Average } t = 300.00 + 200.00 / 2.00 = 250.00 \text{ mm}$$

% of distribution steel = 0.30 [250.0-100.0/450.0-100] x 0.1 = 0.24

$$\text{Ash offered} = 0.24 x 250 x 1000 / 100 = 650.00 \text{ mm}^2$$

Area of steel on each-face = 325.00 mm<sup>2</sup>

Space of 8 mm Ø bar = 1000.0 x 00.785 x 8<sup>2</sup> / 325 = 155. mm

**∴ offer 8.00 mm Ø bars @ 150.00 mm c/c on both faces**

To resist the hoop tension at 2 m below top

$$\text{Ash} = 2.00 x 2286.00 / 5.00$$

$$\text{Ash} = 914.400 \text{ mm}^2$$

Space of 12.00 mm Ø rings = 1000.00 x 113.00 / 914.40/2.00

Space of 12.00 mm Ø rings = 247.00 mm

**∴ offer Space of 12.00 mm Ø rings 240.00 mm c/c in the top 2.00 m height's**

At 3.00 m below's the top

$$\text{Ash} = 3.00 x 2286.00 / 5.00$$

$$\text{Ash} = 1372.00 \text{ mm}^2$$

Space of 12.00 mm Ø rings = 1000.00 x 113.00 / 1372.00/2.00

Space of 12.00 mm Ø rings = 164.700 mm

**∴ offer Space of 12.00 mm Ø rings 160.00 mm c/c in the next 1.00 m height's**

At 4 m below's the top

$$\text{Ash} = 4.00 x 2286.00 / 5.00$$

$$\text{Ash} = 1829.00 \text{ mm}^2$$

Space of 12.00 mm Ø rings = 1000.00 x 113.00 / 1829.00/2.00

Space of 12.00 mm Ø rings = 123.60 mm

**∴ offer Space of 12.00 mm Ø rings 120.00 mm c/c in the next 1.00 m height's**

∴ offer Space of 12.00 mm Ø rings 95.00 mm c/c as found earlier

#### **5 DESIGN OF RING BEAM B3**

The ring beam joins the tank wall through conical dome. The vertical load at the junction of the wall with conical dome is shifted to ring beam B3 by meridional's thrust's in the conical dome. The horizontal's element of the thrust's causes hoop's tensions at the joint. The ring beam is offered to take up this hoop's tensions refer fig 2 the load W transmitted through tank wall at the top of conical dome consist of the following

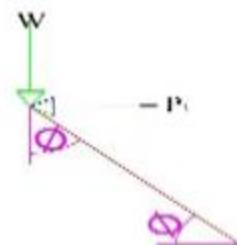


Fig. 2: Load Transmitted

1) Load's of top dome =  $T_1 \sin \theta_1 = 30938.00 \times 0.4824.00 = 14924.00 \text{ N/m}$

2) Load's due to the ring beam B1 =  $0.360 \times (0.40 - 0.20) \times 1.00 \times 25000.00 = 1800.00 \text{ N/m}$

3) Load's due to tank wall =  $5.00 (0.20 + 0.30 / 2.00) \times 1.00 \times 25000.00 = 31250.00 \text{ N/m}$

4) Self 'sload's of beam B3 =  $(1.00 - 0.30) \times 0.60 \times 25000.00 = 10500.00 \text{ N/m}$

Entire  $W = 14924.0 + 1800.0 + 31250.0 + 10500.0 = 58474.0 \text{ N/m}$

Angle of conical domes wall with Vertical Ø° = 45.00°

$$\sin\theta = \cos\theta = 0.70710 \quad \tan\theta = 1.00$$

$$P_w = W \times \tan\theta = 58474.00 \times 1.00$$

$$P_w = 58474.00 \text{ N/m}$$

$$P_w = w \times h \times d = 9800.00 \times 5.00 \times 0.600$$

$$P_w = 29400.00 \text{ N/m}$$

∴ hoops tension's in the rings beams is given by

$$P_3 = (P_w + P_w) \times D/2$$

$$P_3 = (58474.00 + 29400.00) \times 14.00 / 2.00$$

$$P_3 = 615118.00 \text{ N}$$

This to be resisted entirely by steel hoops, the area of which is

$$A_{sh} = P_3 / \text{permissible Stress's}$$

$$A_{sh} = 615118.00 / 150.00 = 4100.00 \text{ mm}^2$$

$$\text{Number of 28 mm } \varnothing \text{ bars} = 4100.00 / 615.75.00$$

∴ offer 7 rings of 28  $\varnothing$  bars

$$\text{Actual } A_{sh} = 0.785 \times 28^2 \times 7$$

$$A_{sh} = 4310.26 \text{ mm}^2$$

Stress's in equal section =  $P_3 / d \times h + (m-1) \times A_{sh}$  offered

$$\text{Stress's in equal section} = 615118.00 / (1000.00 \times 600.00) + (13.0 - 1.0) \times 4310.260$$

$$\text{Stress's in equal section} = 00.940 \text{ N/ mm}^2 < 1.200 \text{ N/mm}^2 \text{ safe}$$

8.00 mm  $\varnothing$  distribution's bars offered in the wall @ 150.00 mm c/c should be taken to rounded off the above ring acts as stirrups

## 6 DESIGN OF TAPERING DOME

1) Meridional's thrusts; - the weight's of water (fig 3)

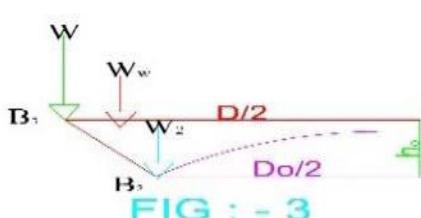


Fig. 3: Forces at conical section

$$W_w = \frac{\pi}{4} D^2 * h + \frac{\pi}{12} * h o (D^2 + Do^2 + D * Do) - \frac{\pi}{3} h * 2^2 (3R^2 - h)$$

$$W_w = 4392368.00 \text{ N}$$

Suppose the thickness of conical slab be 400.00 mm

∴ Total Self Weight Ws is given by

$$W_s = [\pi * \left(\frac{D+Do}{2}\right) x l x to] x \gamma c$$

$$W_s = 25000.00 \times 3.140 (14.00 + 10.00 / 2) \times 2.820 \times 0.40$$

$$W_s = 1066131.00 \text{ N}$$

$$\text{Weight W at B3} = P_w$$

$$W = 58474.00 \text{ N/m}$$

∴ Perpendicular load W2 per metre run is given by

$$W_2 = \frac{\pi * D * W + W_w * W_s}{\pi D o}$$

$$W_2 = \frac{\pi \times 14.00 \times 58474 + 4392368.00 + 1066131}{\pi \times 10.00}$$

$$W_2 = 255613.00 \text{ N/m}$$

Meridional's thrusts To in the conical dome

$$To = W_2 / \cos\theta$$

$$To = 255613.00 \times 1.4140$$

$$To = 361437.00 \text{ N/m}$$

$$\text{Meridional's Stress's} = To / b * d$$

$$\text{Meridional's Stress's} = 361437.00 / 1000.00 \times 400.00$$

$$\text{Meridional's Stress's} = 0.900 \text{ N/mm}^2$$

$$0.90 \text{ N/mm}^2 < 1.200 \text{ N/mm}^2 \text{ :- safe}$$

**b) Hoop Tension:** - Fig no 3 Diameter of tapering dome at any height h' above base is  $D' = 10.0 + (14.0 - 10.0 / 2)h' = 10.0 + 2h'$

Intensity's of water pressure  $P = (5.00 + 2 - h') \times 9800.00 \text{ N/m}^2$

Hoop's tensions P'o is given by

$$P'o = (P / \cos\theta + q \times \tan\theta) D' / 2$$

$$P'o = [(5.00 + 2 - h') \times 9800.00 \times 1.4140 + 1000 \times 1 \{10.0 + 2h' / 2.0\}]$$

$$P'o = 535075.00 + 37720.00 \times h' - 13859h'^2$$

The value of P'o at h'=0, h'=1 and h'=2 are tabulated below

h'	Hoop Tensions
0	535075.00 N
1	558936.00 N
2	555079.00 N

Table .I: Hoop tension

For Maxima,  $d P'o / dh' = 0$

$$37720.00 - 2.0 \times 13859.00 \times h' = 0$$

$$\text{From Which } h' = 1.3610 \text{ m}$$

$$\text{Max } P'o = 535075.0 + 37720.0 \times (1.3610 - 13859.0) \times (1.3610)^2$$

$$\text{Max } P'o = 560739.00 \text{ N}$$

**C) Design of Walls :-**

$$\text{Meridional's Stress's} = 0.900 \text{ N/mm}^2$$

$$\text{Max Hoop Stress's} = 560739.00 \text{ N}$$

Whole of Which is to be resisted by steel

$$As = \text{max hoop Stress's} / \text{permissible Stress's}$$

$$As = 560739.00 / 150.00 = 3738.00 \text{ mm}^2$$

$$\text{Area of Each-face} = 3738.00 / 2.00 = 1869.00 \text{ mm}^2$$

$$\text{Space of 16.0 mm } \varnothing \text{ bars} = 1000.0 \times 0.7850 \times 16.00^2 / 100.0 = 107.05$$

**Space of 16.0 mm  $\varnothing$  hoops @ 100.0 mm c/c on each-face**

$$\text{Actual } A_{sh} = 1000.0 \times 0.7850 \times 16.00^2 / 100.0 = 2010 \text{ mm}^2$$

$$\text{Ash offered} = 2010.00 \text{ mm}^2$$

Max tensile Stress's in composite's section

$$= 560739.00 / (400.0 \times 1000.0) + (13-1) \times 2010.0 \times 2.0$$

$$\text{Max. tensile Stress's in composite section} = 1.3850 \text{ N/mm}^2$$

This tensile Stress's is more than the permissible, value 1.20 N/mm<sup>2</sup>.∴ increase the thickness **420.0 mm**. this will reduce the tensile Stress's to **1.1980 N/mm<sup>2</sup>** :- safe

In the meridional direction offer reinforcement @

$$\{0.30 - [420.0 - 100.0 / 420.0 - 100.0] \times 0.10\} = 0.210\%$$

$$Asd = 0.21 \times 4200 = 882.00 \text{ mm}^2$$

$$\text{Asd on each-face} = 882.00 / 2.00 = 441.00 \text{ mm}^2$$

$$\text{Space of 10 mm } \varnothing \text{ bars} = 1000.0 \times 0.7850 \times 10^2 / 441.0 = 178 \text{ mm}$$

-: offer 10.00 mm bars @ 175.00 mm c/c on each-face clear- cover 25.00mm

## 7 DESIGN OF LOWEST DOME

Lowest dome develops compressive Stress's both meridionally's as well as hoop's, due to weight's of water buoyed by it and also due to its own weight

$$R2 = 8.610 \text{ m}; \sin\theta = 0.58070; \cos\theta = 0.81410$$

**Weight of Water Wo of the dome is given by**

$$Wo = \left[ \frac{\pi}{4} D_o^2 H_o - \frac{\pi}{3} h^2 (3R2 - h^2) \right] \times w$$

$$Wo = \left[ \frac{\pi}{4} 10^2 \times 7 - \frac{\pi}{3} 1.6^2 (3 \times 8.610 - 1.60) \right] \times 9800.00$$

$$Wo = 4751259.00 \text{ N}$$

Let the thickness of bottom dome be 250.00 mm

$$\text{Self-Weight} = 2 \times 3.140 \times R2 \times h2 \times t2 \times \gamma_c \quad [\gamma_c = 25000]$$

$$\text{Self-Weight} = 2 \times 3.140 \times 8.610 \times 1.60 \times 0.250 \times 25000.00$$

$$\text{Self-Weight of dome} = 540982.00 \text{ N}$$

$$\text{Total Weight WT} = W_w + Wo$$

$$\text{Total Weight WT} = 540982.00 + 4751259.00$$

$$\text{Total Weight WT} = 5292241.00 \text{ N}$$

Meridional's Thrust T2 = WT/3.140 x Do x sinθ

$$\text{Meridional's Thrust T2} = 5292241.0 / 3.140 \times 10.0 \times 0.58070$$

$$\text{Meridional's Thrust T2} = 290093.00 \text{ N/m}$$

$$\text{Meridional's Stress's} = T2 / b * d$$

$$\text{Meridional's Stress's} = 290093.00 / 250.00 \times 1000.00$$

$$\text{Meridional's Stress's} = 1.160 \text{ N/mm}^2$$

$$1.160 \leq 1.200 \text{ N/mm}^2 \quad \text{:- safe}$$

$$\text{Intensity of loading per unit area} = P2 = WT / 2 \times 3.140 \times R2 \times h2$$

$$P2 = 5292241.00 / 2.00 \times 3.140 \times 8.610 \times 1.60$$

$$P2 = 61142.00 \text{ N/m}^2$$

$$\text{Max. hoop Stress's at Centre of dome} = P2 * R2 / 2 * t2$$

$$\text{Max. hoop Stress's at Centre of dome} = 61142.0 \times 8.61 / 2 \times 0.25$$

$$\text{Max. hoop Stress's at Centre of dome} = 1.050 \text{ N/mm}^2$$

**Safe**

$$\text{Area of minimum steel} = 0.3 - [250-100/450-100] \times 0.1 = 0.26 \text{ %}$$

$$As = 0.260 \times 2500.00 = 650.00 \text{ mm}^2 \text{ in each direction}$$

$$\text{Space of 10 mm } \varnothing \text{ bars} = 100.0 \times 0.785 \times 10^2 / 650 = 121.00 \text{ mm}$$

-: offer Space of 10.00 mm  $\varnothing$  bars @ 120.0 mm c/c in both directions also offer 16.0 mm  $\varnothing$  meridionals bar @ 100.0 mm c/c near water face. For 1.0 m length to take care of the continuity effect the thickness of the dome may be increased from 250.0 mm to 280.0 mm gradually in 1.0 m length

## 8 DESIGN OF LOWEST CIRCULAR BEAM B2:-

Inner thrust from tapering dome =  $To \times \sin\theta$

Inner thrust from tapering dome =  $361437.0 \times 0.7070$

**Inner thrust from tapering dome = 255613.0 N/m**

outer thrusts from bottom dome =  $T2 \times \sin\theta$

outer thrusts from bottom dome =  $290093.0 \times 0.81410$

**outer thrust from bottom dome = 236165.00 N/m**

Net Inner thrusts = Inner - Outer

Net Inner thrusts =  $255613.00 - 236165.00$

**Net Inner thrust = 19448.00 N/m**

Hoop compression in beam =  $19448.00 \times 10 / 2 = 97240.00 \text{ N}$

**Suppose the sizes of beams be 600.0 mm x 1200.0 mm**

Hoop Stress's =  $97240.0 / 600.0 \times 1200.0 = 0.1350 \text{ N/mm}^2$

**0.1350 N/mm<sup>2</sup> ≤ 1.20 N/mm<sup>2</sup> Safe**

Vertical load on beam per metre run =  $To \times \sin\theta + T2 \times \sin\theta$

Vertical load on beam per metre run =  $255613 + 290093 \times 0.5807$

**Vertical load on beam per metre run = 424070 N/m**

Self-weight of beam =  $b \times d \times \gamma_c$

Self-weight of beam =  $0.6 \times 1.20 \times 1 \times 25000.00$

**Self-weight of beam = 18000.00 N/m**

The load on beam =  $W = \text{vertical loading} + \text{self-weight} = 424070.00 + 18000.00 = 442070.00 \text{ N/m}$

**The loading on beam = W=442070.00 N/m**

Let,s us support the beam on 8.00 similarly spaced column at a mean radii of lowest curved beam R = 5.00 m  $2\theta = 45^\circ$ ;  $\theta = 22.5^\circ$

$C1 = 0.0660$ ;  $C1 = 0.0300$ ;  $C1 = 0.0050$  [IS CODE TABLE 20.1 ]

$\theta_m = 9.50^\circ$

$MO = \text{SUPPORT's MOMENT's B.M -VE} = C1 \times WR^2 \times 2\theta$

$MO = 0.066 \times 442070.0 \times 5.0^2 \times 0.7580$

**MO = 572882 Nm**

Extreme + ve B.M at support =  $Mc = C2 \times WR^2 \times 2\theta$

$Mc = 0.030 \times 442070 \times 5^2 \times 0.785$

**Mc = 260401 Nm**

Extreme To0rsional moment  $Mm = C3 \times WR^2 \times 2\theta$

Extreme Torsional moment  $Mm = 0.005 \times 442070 \times 5^2 \times 0.785$

**Extreme Torsional moment Mm = 43400 Nm**

**For M-20 concrete [ IS 456:2000]**

$\sigma_{cbc} = 7.00 \text{ N/mm}^2$  HYS'D bars  $\sigma_{st} = 150.00 \text{ N/mm}^2$

We have  $K = 0.3780$ ;  $j = 0.8740$ ;  $R = 1.1560$

$$\text{-: effective depth} = \sqrt{\frac{572882 \times 1000}{600 \times 1.156}}$$

$$\text{effective depth} = 909.00 \text{ mm}$$

-: keep total depth = 1200.00 mm from shear point of view

suppose  $d = 1140 \text{ mm}$

Max shear force at support,  $Fo = W \times R \times \theta$

$$Fo = 442070.00 \times 5.00 \times 3.14/8$$

$$F_o = 868002.00 \text{ N}$$

SF at any point is given by  $F = WR(\Theta - \phi)$

At  $\phi = \phi_m$ ;  $F = 442070 \times 5 \times (22.50^\circ - 9.5^\circ) \times 3.14 / 180$

$$F = 501512.00 \text{ N}$$

BM at the point of extreme torsional's moment's

$$\phi = \phi_m = 9.5^\circ \text{ is given by}$$

$$M\phi = WR^2 (\Theta \times \sin\phi + \Theta \times \cot\Theta \times \cos\phi - 1)$$

$$M\phi = 442070 \times 5^2 (\Pi/8 \times \sin 9.5^\circ + \Pi/8 \times \cot 22.5^\circ \times \cos 9.5^\circ - 1)$$

$$M\phi = -1421.00 \text{ Nm (sagging)}$$

At the support  $\phi = 0$

$$M_o = WR^2 (\Theta - \phi) = 0$$

$$\text{At mid span } \phi = \Theta = 22.5^\circ = \frac{\pi}{8} \text{ radians}$$

$$M\phi = WR^2 (\Theta \times \cos\Theta - \Theta \frac{\cos\phi}{\sin\phi} \sin\phi) = 0$$

At the support  $M_o = WT$

$$M_o = 572882 \text{ Nm}$$

At the Mid span

$$M_c = 260401 \text{ Nm sagging +ve}$$

At the point of max. torsion ( $\phi = \phi_m = 9.5^\circ$ )

$$M\phi = 1421 \text{ Nm}$$

$$M_m = 43400 \text{ Nm}$$

### Foremost and Longitudinal Reinforcements

#### a) Section at point of extreme torsion

$$T = M_{max} = 43400 \text{ Nm}; M\phi = M = 1421 \text{ Nm}$$

$$M_{e1} = M + MT$$

$$MT = 76588 \text{ Nm}$$

$$M_{e1} = 1421 + 76588 = 78009 \text{ Nm}$$

$$A_{st1} = 78009 \times 100 / 150 \times 0.874 \times 1160$$

$$A_{st1} = 513 \text{ mm}^2$$

$$\text{No. of 25 mm } \phi \text{ bars} = 513 / 491 = 1.05$$

Let us offer a minimum of 2 bars

Since  $MT > M$

$$M_{e2} = MT - M$$

$$M_{e1} = 76588 - 1421$$

$$A_{st2} = \frac{75167 \times 1000}{150 \times 0.874 \times 1160}$$

$$A_{st2} = 494.30 \text{ mm}^2$$

Number of 25 mm bar = 1 offer minimum of 2 bar at the point of extreme torsion, offer 2 -25 mm  $\phi$  bar each at top and bottom

#### b) Section at extreme hogging B.M (support)

$$M_o = 572882 \text{ Nm} \quad M_o = 0$$

$$A_{st} = \frac{572882 \times 1000}{150 \times 0.874 \times 1140}$$

$$A_{st} = 3767 \text{ mm}^2$$

$$\text{No. of 25 mm } \phi \text{ bar} = \frac{3767}{\pi \times 25^2 / 4} = 8$$

Then Offer 6 nos of 25 mm  $\phi$  bars in one layer and 2 layer in the 2<sup>nd</sup> layer offer at top of the section near support

#### c) Section at max. sagging B.M (mid span)

$$M_c = 260401.00 \text{ Nm} \quad M_c = 0$$

For positive B.M, Steel will be to the other face, Where Stress's in steel ( $\sigma_{st}$ ) can be taken as 190.00 N/mm<sup>2</sup>. The

Constant for M-20 concrete having  $\sigma_{cbc} = 7.00 \text{ N/mm}^2$ ,  $m = 13.00$  will be

$$K = 0.324 : j = 0.892 : R = 1.011$$

$$A_{st} = \frac{260401 \times 1000}{190 \times 0.892 \times 1160}$$

$$A_{st} = 1325.000 \text{ mm}^2$$

$$\text{No. of 25 mm } \phi \text{ bars} = \frac{1325}{\pi \times 25^2 / 4} = 2.7$$

∴ the structure of reinforcements will be follow's at the support, offer 6.0 – 25.0 mm  $\phi$  at top layer and 2.0- 25.0 mm  $\phi$  bar in the 2<sup>nd</sup> layer continue these up to section of extreme torsion at a distance =  $R \times \phi_m = 5.00 \times 0.166 = 0.830 \text{ m}$

$$L_d = 52 \times \phi = 52 \times 25$$

$$L_d = 1300.00 \text{ mm from support}$$

At this point, discontinue 4 bars while the remaining 4 bar similarly offer 4 bar 25.00 mm  $\phi$

At the bottom throughout the length. these bars will take care of both the max positive B.M as well as extreme torsional moment

#### Transverse Reinforcement: -

a) At point of max. torsional moments

At point of max. torsion  $V = 501512.00 \text{ N}$  (step – 8 value obtained)

$$V_e = V + 1.6 \times \frac{T}{b} \quad \text{Where } T = M_m = 43400.00 \text{ Nm}$$

$$b = 600.00 \text{ mm}$$

$$V_e = 501512.00 + 1.60 \times \frac{43400.00}{0.60}$$

$$V_e = 617245.00 \text{ N}$$

$$\tau_{ve} = V_e / b \times d$$

$$\tau_{ve} = 617245.00 / 0.60 \times (1200.00 - 400.00)$$

$$\tau_{ve} = 0.887 \text{ N/mm}^2$$

This is less than  $\tau_{max} = 1.800 \text{ N/mm}^2$

for M-20 concrete (IS 456 : 2000 table 20.8)

$$\tau_c = 0.230 \text{ N/mm}^2$$

Since  $\tau_{ve} > \tau_c$ , Shear reinforcement is necessary

The area of cross section  $A_{sv}$  of the stirrups is given by

$$A_{sv} = \frac{T \times S_v}{b \times d \times \sigma_{sv}} + \frac{V \times S_v}{2.5 \times d \times \sigma_{sv}}$$

$$\text{Where } b_1 = 600 - (40 \times 2) - 25 = 495.00 \text{ mm}$$

$$d_1 = 1200 - (40 \times 2) - 25 = 1095.00 \text{ mm}$$

$$\frac{A_{sv}}{S_v} = \frac{43400}{495 \times 1095 \times 150} + \frac{501512}{2.5 \times 1095 \times 110} = 1.755$$

Minimum Transverse's reinforcements is governed by

$$\frac{A_{sv}}{S_v} \geq \frac{\tau_{ve} - \tau_c}{\sigma_{sv}} \times b$$

$$\frac{A_{sv}}{S_v} = \frac{0.887 - 0.23}{150} \times 600.00$$

$$\frac{A_{sv}}{S_v} = 2.628$$

$$\therefore \text{Depth } \frac{A_{sv}}{S_v} = 2.628$$

Using,  $s = 12.0 \text{ mm } \phi$  4 legged stirrups,  $A_{sv} = 4 \times \frac{\pi}{4} \times 12^2 = 452.0 \text{ mm}^2$

$$\text{Or } S_v = \frac{452}{2.628} = 172.00 \text{ mm}$$

But, the space should not exceed the least of  $X_1$ ,  $\frac{X_1+Y_1}{d}$  and 300 mm

-: offer 12 mm Ø 4 legged stirrups @ 170 mm c/c

### b) At the point of max. shear (support )

At support ;  $F_o = 868002$  N

$$\tau_v = \frac{868002}{600 \times 1160} = 1.25 \text{ N/mm}^2$$

At Support ,

$$\frac{100 \times As}{bd} = \frac{100 \times 8 \times 491}{600 \times 1160} = 0.564$$

$\tau_c = 0.310$  N/mm<sup>2</sup> , Shear Reinforcement is necessary

$$V_c = 0.310 \times 600.00 \times 1160.00 = 215760.00$$

$$V_s = F_o - F_s = 868002.00 - 215760.00 = 652242.00 \text{ N}$$

Space of 10.0 mm Ø 4 legged stirrups having  $As_v = 314.0 \text{ mm}^2$

Given by

$$S_v = \frac{\sigma_{sv} \times As_v \times d}{V_s}$$

$$S_v = \frac{150 \times 314 \times 1160}{652242} = 83.80 \text{ mm}$$

-: offer 12 mm Ø 4 legged stirrups

$$As_v = 4 \times \frac{\pi}{4} \times 12^2 = 452.39 \text{ mm}^2 \text{ at space}$$

$$S_v = \frac{150 \times 452.39 \times 1160}{652242} = 120.0 \text{ mm}$$

### C) At Mid span

At the mid span, SF is Zero -: offer minimum shear reinforcement given by

$$\frac{As_v}{bS_v} \geq \frac{0.4}{f_y}$$

$$\frac{As_v}{S_v} = \frac{0.4 \times b}{f_y} \quad \text{for HYSD bar } f_y = 415 \text{ N/mm}^2$$

$$\frac{As_v}{S_v} = \frac{0.4 \times 600}{415} = 0.578$$

Choosing 10 mm Ø 4 legged stirrups  $As_v = 314 \text{ mm}^2$

$$S_v = \frac{314}{0.578} = 543 \text{ mm}$$

Max permissible space  $0.75d = 0.75 (1200-40) = 870$  or 300 mm

Whichever is less -: offer 10 mm Ø 4 legged stirrups @ 300 mm c/c

Side Reinforcement: -

Since the depth is more than 450 mm, offer side face reinforcement @ 0.1 %

$$At = \frac{0.1}{100} \times 600 \times 1200 = 720 \text{ mm}^2$$

Offer 3-16 mm Ø bar on each-face having total At = 6 x 201 = 1206 mm<sup>2</sup>

## 9 DESIGN OF COLUMNS

The tank is supported on 8 columns symmetrically placed on a circle of 10 m mean diameter. Height of staging above ground level is 16 m let us divide this height into four panels each of 4 m height. Let column connected to raft foundation by means of a ring beam, the top of which is offered at 1 m below the ground level, so that the actual height of bottom panel is 5 m.

### A ) Vertical loads on columns :-

$$1) \text{ Weight of water} = W_w + W_o = 4392368 + 4751259 = 9143627 \text{ N}$$

2 ) Weight of tank :-

$$i) \text{ Weight of top dome + cylindrical walls} = 58474 \times \pi \times 14 = 2571821 \text{ N}$$

ii ) Weight of tapering dome =  $W_s = 1066131 \text{ N}$

iii ) Weight of lowest dome = 540982 N

iv ) Weight of lowest ring beam =  $18000 \times \pi \times 10 = 565487 \text{ N}$

Entire weight of tank = i + ii + iii + iv

**Entire weight of tank = 4744421 N**

Total Superimposed load = weight of water + Total weight of tank

Total Superimposed load = 9143627 + 4744421

**Total Superimposed load = 13888048 N**

**Load Per column = 13888048 / 8 = 1736000 N**

Supposing the column be 700.00 mm diameter

Weight of column per metre height =  $\frac{\pi}{4} \times 0.7^2 \times 1 \times 25000 = 9620 \text{ N}$

**Supposing the bracing be of 300 mm x 600 mm size**

$$\text{Length of Each Brace} = L = R \times \frac{\sin \frac{2\pi}{n}}{\cos \frac{\pi}{n}} = 5 \times \frac{\sin \frac{2\pi}{8}}{\cos \frac{\pi}{8}} = 3.83 \text{ m}$$

Clear length of each brace =  $3.830 - 0.70 = 3.130 \text{ m}$

**Weight of Each brace =  $0.3 \times 0.6 \times 3.13 \times 25000 = 14085 \text{ N}$**

-: total Weight of column just above each brace is tabulated below

Brace GH:

$$W = (134720 + 11760 + 33060) + 4 \times 9620 = 1774480.00 \text{ N}$$

Brace EF:

$$W = (134720 + 11760 + 33060) + 8 \times 9620 + 14085 = 1827045.00 \text{ N}$$

Brace CD:

$$W = (134720 + 11760 + 33060) + 12 \times 9620 + 2 \times 14085 = 1879610.00 \text{ N}$$

Bottom of column:

$$W = (134720 + 11760 + 33060) + 17 \times 9620 + 2 \times 14085 = 1941795.00 \text{ N}$$

Wind loads

Intensity of wind pressure = 1500.00 N/m<sup>2</sup>

Suppose take a factor of 0.7 for section in circular in plan

Wind load on tank, domes and ring beam

$$= [(5 \times 14.4) + (14.2 \times 2/3 \times 1.9) + (2 \times 12.8) + (10.6 \times 1.21)] \times 1500 \times 0.7 = 134720 \text{ N}$$

This may be assumed to act at about 5.7 m above the bottom of ring beam.

wind load on each panel of 4 m height of column =  $(4 \times 0.7 \times 8) \times 1500 \times 0.7 + (0.6 \times 10.6) \times 1500$

wind load on each panel of 4 m height of column = 33060 N

wind load at the top end of top panel =  $0.5 \times 23520 = 11760 \text{ N}$

wind load are shown in fig below

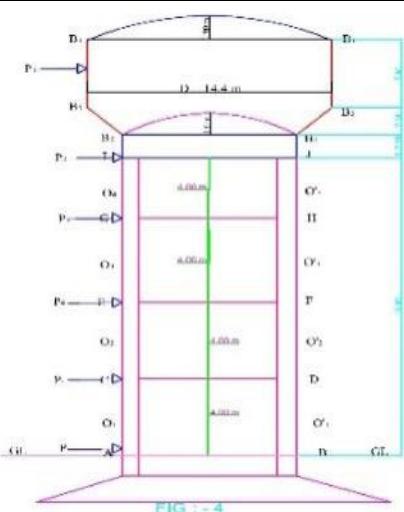


Fig. 4:wind pressure on intz tank

The point of contra flexure O1 O2 O3 and O4 are assumed to be at the mid height of each panel. the shear forces Qw and moment Mw due to wind at these planes are given below

Level	Shear Force Qw (N)	Moment Mw (Nm)
O4	146480.00	1060860.00
O3	179540.00	1712900.00
O2	212600.00	2497180.00
O1	245660.00	3418930.00

Table. 2: Shear force and moment

The Axial thrust  $V_{max} = 4 \times M_w / n \times D_o$

The Axial thrust  $V_{max} = 4 \times M_w / 8 \times 10 = 0.05 M_w$

The Axial thrust  **$V_{max} = 0.05 M_w$**  in the farthest leeward column the shear force

$S_{max} = 2 \times Q_w / n = 2 \times Q_w / 8 = 0.25 Q_w$

**$S_{max} = 0.25 Q_w$**

In the column on the bending axis at each of the above levels and the bending moment

**$M = S_{max} \times h/2$  in the column are tabulated below**

Table. 3: column forces and bending

Level	$V_{max} = 0.05$	$S_{max} = 0.25$	$M = S_{max} \times h/2$
O4	53040	36620	73240
O3	85650	44895	89770
O2	124860	53150	106300
O1	170950	61420	153550

Table. 4: Axial force and moment

Level	Farthest leeward column		Column on bending axis	
	Axial (N)	$V_{max}(N)$	Axial (N)	$M (Nm)$
O4O'4	1774480	53040	1774480	73240
O3O'3	1827045	85650	1827045	89770
O2O'2	1879610	124860	1879610	106300
O1O'1	1941795	170950	1941795	153550

The farthest leeward column will be endangered to the superimposed axial load plus  $V_{max}$  given above. The column on the bending axis on the permissible Stress's in the material may be enlarged by 33.33% for the farthest leeward column the axial thrust  $V_{max}$  due to wind load is less than even 10 % of the superimposed axial load. :- the effect of wind is not critical for the farthest leeward column however, column is situated on the bending axis need to be considered to see the effect of extreme B.M of 153550.00 Nm . due to wind along with the superimposed axial load of 1941795.00 N at the lowest panel .

Use M-20 Concrete For Which

$\sigma_{cbc} = 7.00 \text{ N/mm}^2 \quad \sigma_{cc} = 5.00 \text{ N/mm}^2$  [ IS 456:2000 ]

For Steel  $\sigma_{st} = 230.00 \text{ N/mm}^2$

All these three can be increased by 33.33%. When considering action. Diameter of column = 700.00 mm

Use 13 bars of 28 mm  $\varnothing$  at an effective cover of 40 mm

$$A_{sc} = \frac{\pi}{4} \times 28^2 \times 13$$

$$A_{sc} = 8482 \text{ mm}^2$$

$$\text{Equal area of column} = \frac{\pi}{4} \times 700^2 \times (13 - 1) \times 8482$$

$$\text{Equal area of column} = 486629 \text{ mm}^2$$

$$\text{Equal moment of inertia} = \frac{\pi}{64} d^4 + (n - 1) \frac{A_{sc}}{8} \times d'^2$$

Where  $d = 700.00 \text{ mm}$   $d' = 700.00 - 40.0 \times 2.0 = 620.00 \text{ mm}$

$$I_c = \frac{\pi}{64} 700.00^4 + (13.0 - 1.0) \frac{8482.00}{8.00} \times 620.00^2$$

$$I_c = 1.667660000 \times 10^{10} \text{ mm}^4$$

Direct Stress's in column =  $\sigma_{cc}' = 1941795.0 / 486629.0 = 3.990 \text{ N/mm}^2$

Bendings Stress's in column =  $\sigma_{cbc}' = \frac{153550 \times 1000}{1.66766 \times 10^{10}} = 3.22 \text{ N/mm}^2$

For the safetys of column's, we have the condition

$$\frac{\sigma_{cc}'}{\sigma_{cc}} + \frac{\sigma_{cbc}'}{\sigma_{cbc}} \leq 1$$

$$\frac{3.99}{3.22} + \frac{3.22}{1.33 \times 7} \leq 1$$

**0.95 < 1**

**safe**

Use 10.00 mm  $\varnothing$  wire rings of 250.00 mm c/c to ties up the mains reinforcements. Since the columns are of 700.00 mm diameters rise the width of curved beam B2 from 600.00 mm to 700.00 mm

## 10 DESIGN OF BRACES

The bending moment  $m_1$  and extreme value in a brace is governed by step 9

$$\tan(\theta + \frac{\pi}{8}) = \frac{1}{2} \cot \theta$$

We get  $\theta = 24.80^\circ$

$$m_1 = \frac{Qw_1 x h_1 + Qw_2 h_2}{n \sin \frac{2\pi}{8}} \cos^2 \theta \sin(\theta + \frac{\pi}{n})$$

For the lowest junction C  $h_1=5.00$  m and  $h_2 = 4.00$

$$m_1 = \frac{245660 x 5 + 212600 x 4}{8 x \sin \frac{2\pi}{8}} \cos^2 24.8^\circ \sin(24.8^\circ + \frac{\pi}{8})$$

**m<sub>1max</sub> = 222540.00 Nm**

The max. shear force  $S_{bmax}$  in a brace is given by for  $\theta = \frac{\pi}{8}$

$$S_{bmax} = \frac{245660 x 5 + 212600 x 4}{3.93 x 8 x \sin \frac{2\pi}{8}} 2 \cos^2 \frac{\pi}{8} \sin \frac{2\pi}{8}$$

**S<sub>bmax</sub> = 112870.00 N**

for  $\theta = \frac{\pi}{8}$  the value of  $m_1$  is given by

$$m_{1max} = \frac{245660 x 5 + 212600 x 4}{8 x \sin \frac{2\pi}{8}} \cos^2 \frac{\pi}{8} \sin(\frac{\pi}{8} + \frac{\pi}{8})$$

$m_{1max} = 221786.00$  Nm

Twisting's moments at  $\theta = \frac{\pi}{8}$  is  $M' = 0.05$  m $\text{Nm}$

$$M' = 0.05 \times 221786 = 11090.00 \text{ Nm}$$

Thus, the bracing will be exposed to a combination of max. shear forces and a twisting moments when wind blow parallel to it  $= \frac{\pi}{8}$

Use M-20 Concrete For Which

$$\sigma_{cbc} = 7.00 \text{ N/mm}^2 \quad \sigma_{cc} = 5.00 \text{ N/mm}^2$$

For Steel

$$\sigma_{st} = 230.00 \text{ N/mm}^2$$

$$k = 0.283 \quad j = 0.906 \quad \text{and} \quad R = 0.897$$

$$\text{Depth of NA} = 0.283 \quad d = kd$$

Supposing  $A_{sc} = A_{st} = pbd$  and  $dc = 0.1d$

Equating the moment of equal area about NA  $P = 0.00560$

Since the brace is endangered to both BM as well as twisting moment we have

$$Me_1 = M' + MT$$

$$\text{Where } M' = B.M = 22250.00 \text{ Nm}$$

$$MT = T \left( \frac{1+\frac{D}{b}}{1.7} \right) = MT = 11090.00 \times \left( \frac{1+\frac{700}{300}}{1.7} \right)$$

$$MT = 21745.00 \text{ Nm}$$

$$Me_1 = 222540.00 + 21745.00 = 244285.00 \text{ Nm}$$

In order to find the depth of the section, compare the moment of resisting of the section to the external moment  $b \times n \times c/2 [d.n/3] + (m-1) A_{sc} C' (d-dc) = Me_1$

$$C = 1.330 \times 7.00 = 9.310 \text{ N/mm}^2$$

$$mc = 1.50$$

$$m = 1.50 \times 13.00 = 19.5$$

$$C' = 9.310 (0.230-0.10)/0.2830 = 6.020 \text{ N/mm}^2$$

$$-300.00 \times 0.283 * d \times 9.310/2.0 \times [1.0-0.283/3] d + (19.50-1.0) (0.00560 \times 300.0 * d) * 6.02 (1.0-0.10)$$

$$*d = 244285.00 \times 10^3$$

$$d = 680.00 \text{ mm}$$

Approve  $D = 700.00$  mm so that  $d = 700 - 25 - 10 = 665.00$

$$A_{sc} = A_{st} = pbd = 0.0056 \times 300.0 \times 700.0 = 1176.00 \text{ mm}^2$$

No. of 20 mm Ø bars each at top and bottom

$$100 \times A_{sc} / bd = \frac{100 \times 4 \times 491}{300 \times 700} = 0.94\%$$

Maximum Shear = 112870.00 N

$$V_e = V + \frac{1.6 T}{b}$$

$$V_e = 112870.00 + \frac{1.6 \times 11090}{0.3}$$

$$V_e = 172017.00 \text{ N}$$

$$\tau_{ve} = 172017.00 / 300.00 \times 700.00$$

$$\tau_{ve} = 0.820 \text{ N/mm}^2$$

This is smaller than  $\tau_{cmax}$  but more than  $\tau_c = 0.37$

N/mm<sup>2</sup> :- transverses reinforcements is necessary

$$A_{sv} = \frac{T.S_v}{b_1 \times d_1 \times \sigma_{sv}} + \frac{V.S_v}{2.5 \times d_1 \times \sigma_{sv}}$$

$$b_1 = 230.00 \text{ mm}; d_1 = 630.00 \text{ mm}$$

Using 12.00 mm Ø 2 legged stirrups,  $A_{sv} = 226.00 \text{ mm}^2$

$$A_{sv} = \frac{11090 \times 100}{230 \times 630 \times 230} + \frac{112870}{2.5 \times 360 \times 230}$$

$$A_{sv} / S_v = 0.645$$

Minimum transverse reinforcement is given by

$$\frac{A_{sv}}{S_v} \geq \frac{\tau_{ve} - \tau_c}{\sigma_{sv}} \times b$$

$$\frac{A_{sv}}{S_v} = \frac{0.82 - 0.37}{230} \times 300$$

$$\frac{A_{sv}}{S_v} = 0.05870$$

$$S_v = 350.00 \text{ mm}$$

However, the space should not exceed the least of  $X_1$ ,  $\frac{X_1+Y_1}{d}$  and 300 mm

Where

$$X_1 = \text{Short dim stirrip} = 230.0 + 20.0 + 12.0 = 262.0 \text{ mm}$$

$$Y_1 = \text{Long dime stirrups} = 630.0 + 20.0 + 12.0 = 662.0 \text{ mm}$$

$$\frac{262+662}{4} = 391.00 \text{ mm}$$

:- offer 12.0 mm Ø 2 legged stirrups at 230.0 mm c/c throughout. Since depth of section exceeds 450 mm offer side reinforcement @ 0.1 %

$$A_1 = 0.1/100 \times 300 \times 700 = 210 \text{ mm}^2$$

Offer 2 -10 mm Ø bar at each-face giving total

$$A_L = 4 \times 78.5 = 314 \text{ mm}^2$$

Offer 300 mm x 300 mm haunches at the junction of braces with column and reinforce it with 10 mm Ø bar sizes of various components and geometry

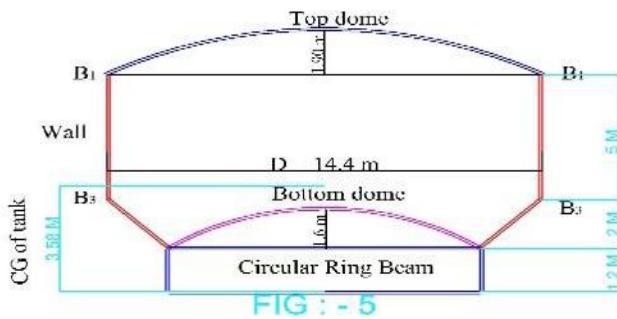


Fig. 5: Component Name

**Sizes of various Components are**

Top Dome 100 thick

Top Ring Beam B1 400 x 360

Cylindrical Wall 200 thick; Bottom Ring Beam B3 700 x 600

Circular Ring Beam B2 600x1200; Bottom Dome 250to 280 thick

Conical Dome 250 thick: Braces 300 x 700

Columns 700 diameter

Constraints of Spring Mass Model

Total weight of water = 9143.627 Kn.

Volume of water = 9143.627 / 9.81 = 932.072 m<sup>3</sup>

Mass of water, m = 932072.06 kg.

Inner diameter of tank, D = 14.00 m.

**For outcome parameters of spring mass model**, an comparable circular container of similar volume and diameter equal to diameter of tank at top level of liquid will be measured.

Let h be the height of equal circular cylinder,

$$\pi (D/2)^2 h = 932.072$$

$$h = 932.072 / [\pi \times (14/2)^2] = 6.05 \text{ m}$$

For  $h/D = 6.05/14 = 0.43$  , [ IS CODE 1893 Part II P.No 10 ]

$$m_i/m = 0.48; m_i = 0.48 \times 932072 = 447394.56 \text{ kg}$$

$$m_c/m = 0.50, m_c = 0.50 \times 932072 = 466036 \text{ kg}$$

$$h_s = 18.20 \text{ m } h_i/h = 0.395; h_i = 0.395 \times 6.05 = 2.38 \text{ m}$$

$$h_{i^*}/h = 0.90; h_{i^*} = 0.9 \times 6.05 = 5.445 \text{ m}$$

$$h_c/h = 0.60; h_c = 0.60 \times 6.05 = 3.63 \text{ m}$$

$$h_{c^*}/h = 0.815; h_{c^*} = 0.815 \times 6.05 = 4.93 \text{ m.}$$

About 55% of liquid mass is excited in impulsive's mode while 43% liquid mass contributes in convective's mode. Sum of impulsive's and convective's mass is 913430.560 kg which is about 2% less than the whole mass of liquid.

weight of empty container + one third weight of staging,

$$m_s = (4744.4210 + 1702.19 / 3) \times (1,000 / 9.81) = 541465.47 \text{ kg.}$$

**Time Period's****Time period of impulsive's mode,**

$$T_i = 2\pi \sqrt{\frac{m_i + m_s}{K_s}} \text{ [IS Code 1893 part 2 pn 16 fig 5]}$$

$$T_i = 2\pi \sqrt{\frac{447394.56 + 541465.47}{132.32 \times 10^5}}$$

$$T_i = 1.70 \text{ sec}$$

**Time period of convective's mode ,**

$$T_c = C_c \sqrt{\frac{D}{g}} \quad [ \text{IS Code 1893 part 2 pn 16 fig 5} ]$$

$$T_c = 3.20 \sqrt{\frac{14}{9.81}} \quad h/D = 0.43 \quad C_c = 3.20$$

$$T_c = 3.82 \text{ sec}$$

**Design Horizontal Seismic Coefficient**

Design horizontal seismic coefficient for impulsive mode ,

$$(Ah)i = \frac{Z \times I}{2 \times R} \left( \frac{Sa}{g} \right)$$

Where, Zone = V I = 1.5 R = 2.5

Z = 0.36 (IS 1893(Part 1): Table 2; Zone V)

$$T_i = 1.70 \text{ sec,}$$

Site has Medium soil,

Damping = 5%,

∴  $(Sa/g)i = 0.9$  (IS 1893(Part 1): Figure 2)

$$(Ah)i = \frac{0.36 \times 1.5}{2 \times 2.5} \times 0.9$$

$$(Ah)i = 0.097$$

**Design horizontal seismic coefficient for convective mode,**

$$(Ah)c = \frac{Z \times I}{2 \times R} \left( \frac{Sa}{g} \right)$$

Where, I = 2.5, R = 2.5

Zone = V Z = 0.36 (IS 1893(Part 1): Table 2; Zone V)

$$T_c = 3.82 \text{ sec}$$

Damping = 5%

$$(Sa/g)c = 0.45 \times 1.75 = 0.787$$

Multiplying factor of 1.75 is used to obtain  $Sa/g$  values for 0.5% damping from that for 5% damping.

$$(Ah)c = \frac{0.36 \times 1.5}{2 \times 2.5} \times 0.787$$

$$(Ah)c = 0.0840$$

**Base-Shear****Base-shear at the lowermost of staging, in impulsive mode,**

$$Vi = (Ah)i (m_i + m_s) g$$

$$Vi = 0.0970 \times (447394.56 + 541465.47) \times 9.81$$

$$Vi = 940.96 \text{ kN-m}$$

Similarly, base shear in convective mode,

$$Vc = (Ah)c m_c g$$

$$Vc = 0.0840 \times 466036.00 \times 9.81$$

$$Vc = 384.03 \text{ Kn}$$

**Whole base-shear at the lowermost of staging by SRSS**

$$V = \sqrt{Vc^2 + Vi^2}$$

$$V = \sqrt{940.96^2 + 384.03^2}$$

$$V = 1016.16 \text{ kN.}$$

**Displacement of tank manual:-**Total displacement =  $H_s/500 = 16000/500 = 32 \text{ mm}$

Table. 5: Displacement Manual

Node	Displacement (mm)
U5	32
U4	24
U3	16
U2	8
U1	0

### III. SOFTWARE DESIGN INTZ WATER TANK

#### Design of intz water tank by using SAP2000 with fixed base:

The seismic presentation of RCC structures earlier and after the application of flexibility and stiffness-based elements method is to be studied in the present project. In this study we are presenting isolation system as a substitute of conventional technique to get improved performance of elevated water tank through the earthquake. This section offers model geometry evidence, including items such as joint coordinates, joint restraints, and element connectivity.

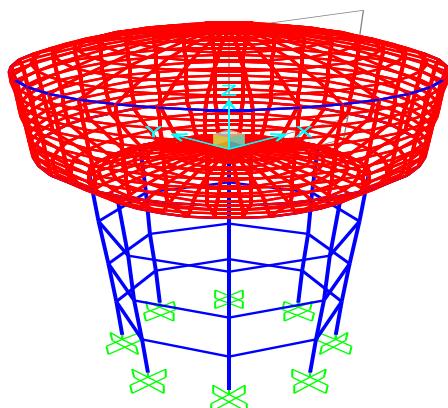


Fig. 6: Finite element model fixed base.

#### Seismic Data:

Seismic Zone: V; Soil Type: Medium soil

Beam Dead Load (UDL): 1.500 KN/m

Live load = 1.5000 KN/m; Water pressure: 0.60 KN/m

#### SAP2000 Analysis

1. Analysis of intz tank is to be performed using Sap2000 for Zone-V.

2. after the analysis is done for fixed base intz water tank is compared with the manual result obtained from manual design and sap2000 design.

3. in this study we have found that base shear and displacement result are equal.

4. but we cannot go for the further manual design of base isolation.

5. So software design by using sap2000 we design structure and compare it with fixed base intz water tank.

#### Total base shear at the bottom of staging by SRSS

$V = 1016.36$  kN.

#### Displacement of tank sap2000: -

Table.6: Displacement sap2000 fixed base.

Table 7: Displacement Base isolation

Node	Displacement (mm)
U5	32
U4	23.4
U3	15.5
U2	8
U1	0

#### Design for intz water tank with base isolation: -

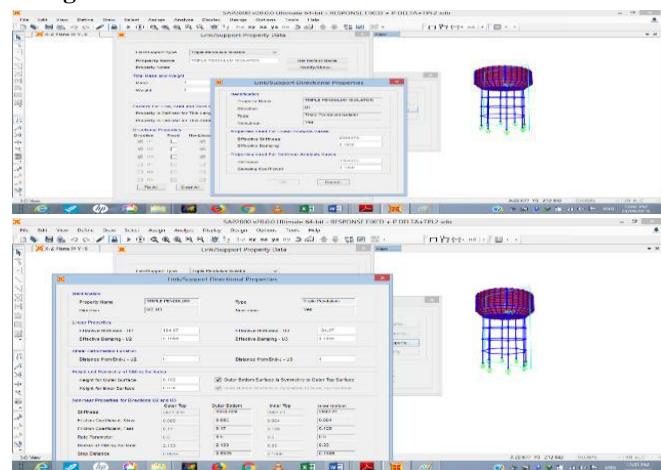


Fig. 7: Finite element model TFPI

#### SAP2000 Analysis

1. Analysis of intz overhead tank is to be performed using Sap2000 with base isolation for Zone-V.

2. we have used the triple friction pendulum isolator at the support at ground level

3. Response analysis is performed for the intz water tank and design is analyzed.

4. So software design by using sap2000 we design structure and compare it with fixed base intz water tank and base isolation.

**Total base shear at the bottom of staging by SRSS**  
 $V = 894.69$  kN.

#### Displacement of tank sap2000 with base isolation: -

Table 7: Displacement Base isolation

Node	Displacement (mm)
U5	5
U4	4
U3	2
U2	1
U1	0

Base shear and displacement analysis are performed with manual and SAP both for both fixed and triple friction pendulum support.

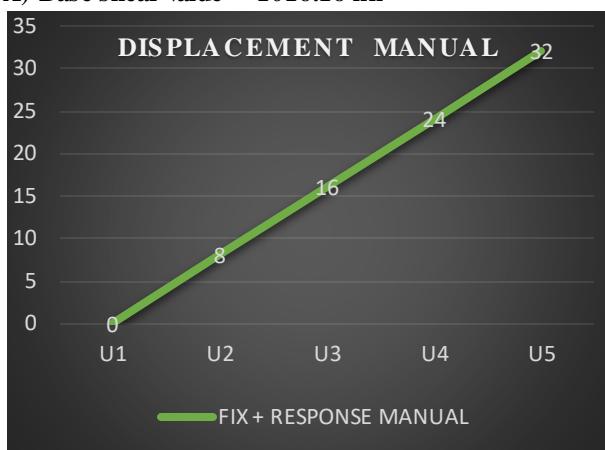
Intz water tank is never been considered under research by researchers with triple friction pendulum

Also, Manual design is not having option for defining base isolation, still we software defined triple pendulum support in sap2000 to compare with manual fixed base elevated tank.

#### IV. RESULT PARAMETER

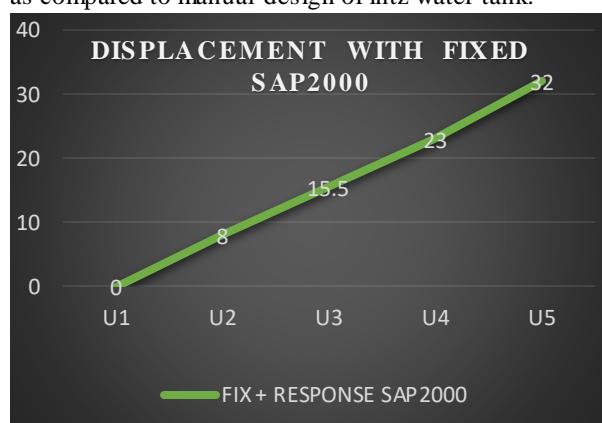
**Parameter for manual:** - In this manual calculation of intz water tank with earthquake resistant parameter. We have Design the parameter of base shear and displacement are as follows

**A) Base shear value = 1016.16 kn**



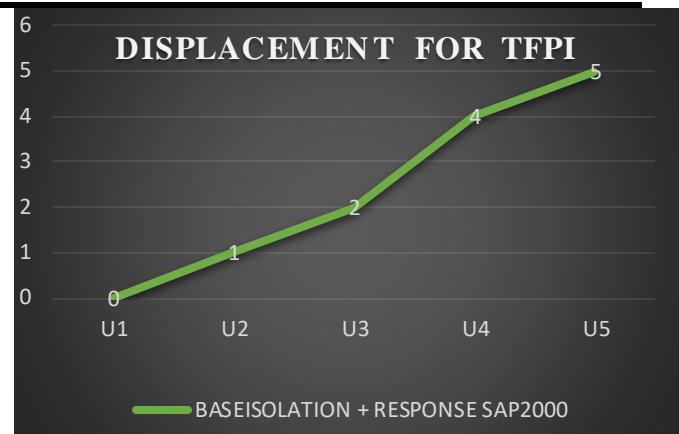
Graph. 1: Displacement

**Result parameter for fixed sap2000:** - In the sap2000 with fixed base of intz water tank. We have compared the design. we have found Same base shear and displacement as compared to manual design of intz water tank.



Graph. 2: Displacement sap2000

**Result parameter for isolated base sap2000:** -In the sap2000 with isolated base of intz water tank. We have found that base shear has been reduced to 12.00 % as compared to manual and base isolation. And also, we have found that there is less displacement as compared to manual with fixed base.

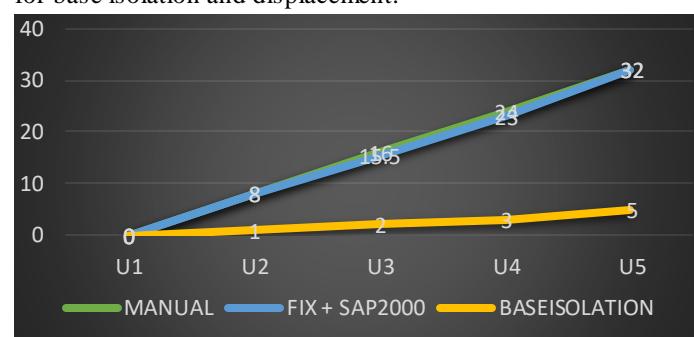


Graph. 3: Displacement base isolation

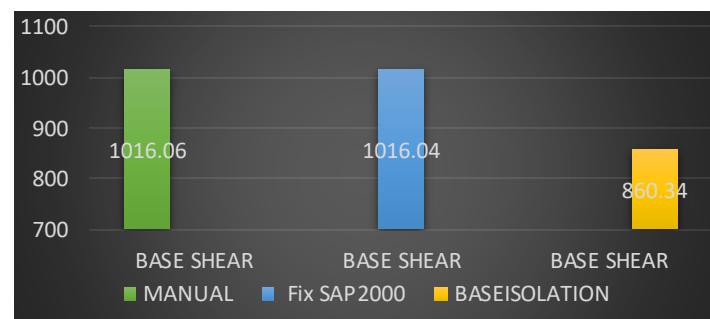
#### V. CONCLUSION

Elevated Water tank is never been considered under research by researchers with triple pendulum isolator. The result which we have obtained for manual fixed base and sap2000 fixed base we found that design for base shear and displacement are quite same. But for the base isolation elevated water tank we have found 2% decrease in base shear and in the displacement up to 90% is decrease with base isolator

Base shear of Zone V because of zone factor same for manual and fixed response reduction factor etc. while considering seismic analysis. And decrease in base shear for base isolation and displacement.



Graph. 4: Comparison of Displacement



Graph. 5: Comparison of Base Shear

**REFERENCES**

- [1] Jain's Sudhisr Ks., Sameers U.Ss., 1990's, —Seismic Designs of Framing Staging for Elevated Waters Tanks,
- [2] Khloud El-Bayoumi's Researcher at Mansoura University, Department. of Civil Engineering, Faculty of Engineering, Egypt
- [3] B.c Pumia book on RCC structure
- [4] IS Code 1893 2002 Part – I and Part -II
- [5] A.k chopra,s book structural dynamic
- [6] IS: 3370 (Part IV) -1967, Design Tables, Codes of Practice's for Concrete's Structure's for the Storage's of liquids.
- [7] IS: 875 (2002) —Codes of Practices for Design's Load || Bureau of Indian Standard, New Delhi.
- [8] IS: 456 (2000) —Plain and Reinforced Concrete- Code's for Practice|| Bureau of Indian Standard, New Delhi.